DEVELOPMENT OF PILE DESIGN METHODOLOGY FOR AN OFFSHORE WIND FARM IN THE NORTH SEA

A.S. Merritt, F.C. Schroeder
Geotechnical Consulting Group LLP, London, UK

R.J. Jardine
Imperial College, London, UK

B. Stuyts, D. Cathie
Cathie Associates SA/NV, Diegem, Belgium

W. Cleverly
GL Noble Denton, Slough, UK

Abstract
This paper describes aspects of the foundation design methodology developed for the Borkum West II offshore wind farm in the German North sea, comprising 40 turbines supported on piled tripods in water depths of approximately 30m. The foundation design evolved during a technical due diligence process, which offered the opportunity to review the site investigation data and cyclic loads, reconsider the effects of cyclic loads on pile resistance, and modify pile lengths and wall thicknesses to mitigate pile tip integrity risk during driving in very dense sands. The re-evaluation of the effect of the design storm concluded that axial pile capacities could fall by up to 25% due to cyclic loading at some turbine locations, but be almost unaffected by cycling at others. The technical review involved a collegiate process that contributed to the development of acceptable foundation designs and mitigated risks relating to pile installation and foundation performance.

1. Introduction

The Borkum West II wind farm is currently being developed by Trianel Windkraftwerk Borkum GmbH in the North Sea, approximately 45km offshore northern Germany (Figure 1). The first phase of this project includes the construction of 40 No. 5MW turbines. The hub height is approximately 90m above sea level, and the rotor diameter is 116m. The turbines are supported in water depths of 26 to 33m by tripod structures designed by Offshore Wind Technologie GmbH. Figure 2 illustrates the general arrangement of the steel tripods, which have an outer footprint diameter of 28m. The legs are founded on 2.48m diameter driven piles and support a central column to which the turbine tower is connected.

Geotechnical engineering was undertaken for the project by Cathie Associates SA/NV (CA), Belgium. The ground investigations, interpretation and foundation designs were performed in accordance with the Standards for Offshore Wind Farms published by the German Federal Maritime and Hydrographic Agency (BSH 2007, 2008). Some aspects of the foundation design methodology, such as analysis of cyclic loading effects, were not covered explicitly by the design codes available at the time, and were developed during the project design phase.

The financing of the project called for a technical due diligence review, which was undertaken by Sgurr Energy Ltd. Geotechnical Consulting Group (GCG) was also engaged to undertake an independent geotechnical design review that developed into a collaborative design process. The principal objective was to identify and mitigate geotechnical risks and develop an agreed foundation design methodology. The review areas included (i) the site investigations and interpretation; (ii) pile installation and driveability; (iii) static pile resistance; and, (iv) storm load characterisation and the effects of cyclic loading on pile axial resistance and lateral loading response.

Figure 1: Location of Borkum West II offshore windfarm

Figure 2: Tripod support structure (www.weserwind.de)

This paper discusses how key geotechnical risks were identified, addressed and mitigated to develop a final design that satisfied the requirements of the certifying authorities and project financiers.

2. Site investigations and interpretation

The site investigations undertaken to characterise the project site and inform selection of the turbine locations and foundation designs were performed in accordance with BSH (2008). A geophysical survey was conducted to investigate the site's geological structure, its degree of lateral variability, the seabed bathymetry and other features. Phased geotechnical surveys comprising CPT/PCPT boreholes with additional soil sampling were undertaken and correlated with the geophysical survey data to determine detailed stratigraphy and geotechnical design parameters for each turbine location. The variations in ground conditions across the site were such that a single CPT/PCPT borehole was required at each turbine location. One such sounding was considered sufficient to characterise conditions over the 28m diameter tripod foundation footprint. The ground conditions consisted principally of dense-to-very dense, fine-to-medium grained sand. The CPT cone resistance (q_c) profiles showed considerable variations in sand state across the site. The q_c values often exceeded 50MPa, and rose above 100MPa at some locations. Layers of medium dense silt and stiff to very stiff clay were identified at some turbine locations.

Axial resistance was the critical design consideration for the tripod piles and these were assessed using the ICP method (Jardine et al, 2005). Good quality site investigations are required to provide:

- continuous CPT/PCPT profiles;
- unit weight measurements;
- soil-steel interface friction angles from ring shear tests;
- yield stress ratio measurements of clays from in-situ tests, laboratory triaxial tests, intact oedometer tests and index properties;
- sensitivity measurements, potentially from intact and remoulded, intact and reconstituted oedometer tests, or index property measurements.

Ideally, cyclic simple shear or triaxial tests should also be performed in cases where cyclic loading action may be important; Jardine (1994).

The site investigations and interpreted geotechnical design parameters represent a potential risk to the satisfactory design and performance of the foundations and were assessed during the design review. The CPT/PCPT boreholes generally provided good quality cone resistance profiles. The q_c profiles were sufficient to characterise the silica sands, and the piezocone profiles allowed differentiation of the sand and silt/clay layers from their pore water pressure responses during penetration. However, at some locations the CPTs had been stopped and the holes drilled out after maximum cone resistances of 50MPa had been reached. In some cases this limited the characterisation of the densest sand layers. The high capacity cones utilised in some soundings recorded q_c values up to 110MPa. Example CPT profiles are shown in Figure 3 for a location (a) where medium-to-very dense sands were encountered above clay/silt layers that extended below the design pile length, and for another location (b) where a thick layer of very dense sand was encountered.

Laboratory tests on pushed samples recovered between CPT strokes provided particle size distributions and index properties that were used in conjunction with the piezocone profiles to designate silt and clay layers. Design values of the sensitivity and Yield Stress Ratio for the identified clay layers were assessed by several methods considering all the available information. This included UU triaxial tests, oedometer tests and index tests together with empirical correlations and parameters derived from the CPT records. As the clay layers were infrequent and relatively thin, only limited data was available to determine clay parameters for specific turbine locations; all of the available laboratory and in-situ test data was reviewed holistically in a "ground model" approach that also addressed the interpreted geological history of the site to ensure that reasonable and consistent parameters were selected.

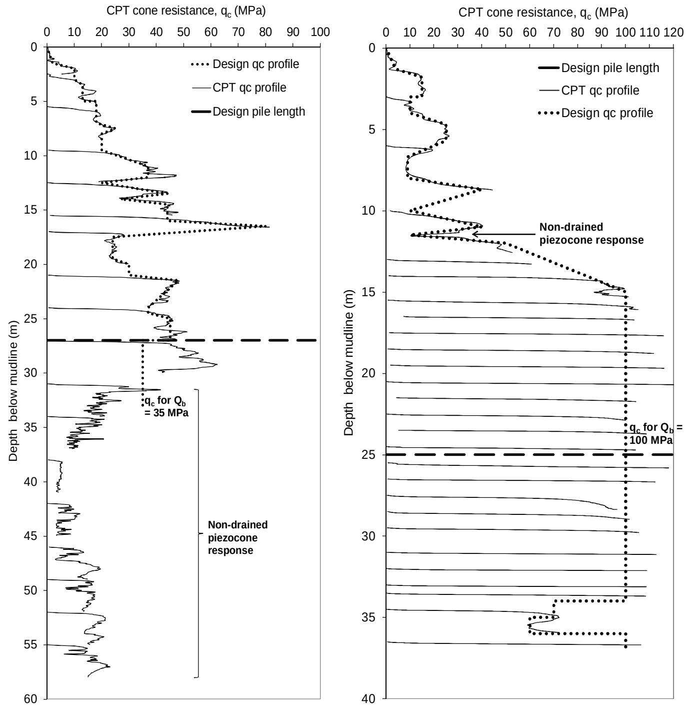

Figure 3: Example CPT profiles (a) sand with clay layers; (b) very dense sand layer

Data from soil-steel interface ring shear testing was available to determine interface friction angles for the sand, silt and clay soils. An average design value was adopted for the sands in the check calculations. The available test data did not indicate any significant variation in interface friction with median grain size over the range studied, as noted in recent research by Ho et al (2010).

Cyclic shear testing was not included in the ground investigations. While such testing is rarely included for piled structures, cyclic loading effects can be significant issue in the design of offshore wind turbine foundations. As described later, cyclic effects were assessed in this case by reference to earlier full scale test programmes on North Sea sands.

Overall, the review confirmed that all the essential geotechnical information required was available to allow an independent interpretation of parameters for check calculations to assess the pile designs.

3. Pile driving risk assessment

When driving piles in very dense sands there is a risk of damage propagating from the pile tips upwards, particularly for piles with high diameter to wall thickness (D/t) ratios. Pile buckling failures have been documented on some offshore oil and gas installations, for example at Goodwyn in NW Australia (Randolph et al, 2005), and Valhall in the North Sea where thick layers of very dense silica sand were encountered at depth (Alm et al, 2004).

The initial pile designs for Borkum West II were based on a 2.48m diameter and 50mm wall thickness (D/t = 50). This configuration was found acceptable for probable pile driving stresses and fatigue design, but the D/t ratio was higher than is typically adopted for oil and gas developments in the North Sea (UK sector), where D/t ratios between 15 and 45 are common and appear to be, on average, around 27 (Jardine, 2009). Piles with D/t ratios of 37 to 62 were successfully driven in dense to very dense sands (q_c of 30 to 60MPa) at the Park Alpha Ventus wind farm close to Borkum West II; see Figure 1. However, several of the Borkum West II turbine locations show q_c values significantly higher than at Alpha Ventus; see Figure 3(b). Some of the higher q_c sand layers at Borkum West II also have greater thicknesses than at Valhall, where piles with D/t = 42 buckled during driving.

Considering the initial pile designs and ground conditions, a qualitative pile driving risk assessment was undertaken to evaluate and mitigate critical pile failure mechanisms identified by MSL Engineering Ltd (2001), namely:

- local buckling at the tip under dynamic driving stresses;
- extrusion buckling related to high lateral soil pressures acting on piles with out-of-roundness imperfections.

To address these risks, the individual turbine locations were categorised based on the CPT profiles, considering the maximum q_c values and the thickness of very dense sand layers. At locations where $q_{c,max}$ < 80MPa, the risk of buckling was considered to be low and the original 50mm wall thickness was retained. The risk of pile tip buckling was considered excessive at locations where $q_{c,max}$ > 80MPa over significant lengths of the profile and the pile wall thickness was increased to 70mm, giving D/t = 35. Overall, pile wall thicknesses were increased at 8 turbine locations. All piles were specified with straight-edged tips, as chamfered pile tips can increase the risk of buckling during driving.

To mitigate the risk of extrusion buckling involving a progressive growth of out-of-roundness during driving, further procedures were specified to (i) avoid pile damage during handling, and (ii) verify the pile ovality remains within specification immediately prior to driving. The pile installation sequence and instrumentation were also designed to provide early warnings of any pile driving problem at high risk locations. Early identification would trigger additional mitigation measures that could be implemented before returning to drive piles at other high risk locations.

4. Assessment of static axial pile resistance

The design review included independent verification of the resistance of the piles. Calculations were performed to check axial resistance and lateral loading response at a number of turbine locations, taking initially the pile lengths and wall thicknesses assigned after completing the pile driving risk assessment. The ground conditions considered covered locations with low, high and typical CPT profiles, and also one where clay layers are interbedded with the sands.

As discussed in Section 2, the ground investigation data was independently interpreted to develop design CPT profiles and geotechnical parameters for each check location. Particular attention was given to sub-dividing CPT profiles finely in order to accurately identify silt/clay layers. The simplified design q_c profiles were defined on a metre-by-metre (or finer) basis as shown in Figure 3.

In relatively uniform cases, the q_c values assigned to calculate base resistance were assessed from the average cone resistance profiles over 1.5 pile diameters above and below the pile tip. However, where the profiles indicated very marked variations, or a weaker soil layer below the pile toe, the design q_c values were reduced to account for the potential impacts on end bearing.

The interpretation of q_c records can significantly influence the axial pile resistance calculated using CPT based design methods. While these interpretations may be partially subjective, careful inspection and interpretation minimises the sensitivity of the calculated pile resistance to the selection made and improves the reliability of the design method.

The static axial pile capacities (in both compression and tension) calculated independently by GCG and CA generally showed good agreement with no systematic differences, and matching within 5 to 10% in both compression and tension at most locations checked. However, at some individual locations more significant differences (up to 20%) followed from different interpretations of the CPT design profiles and geotechnical parameters.

5. Assessment of cyclic loading effects

The design review considered the characterisation of the design storm and the effects of cyclic lateral and axial loading on pile resistance, following an approach similar to that applied in earlier projects including the Clair 1 West of Shetland platform described by Evans et al (2010) and Jardine et al (2011). The Borkum West II project adopted a Eurocode style design criteria as required by German regulations, which differ from the standard offshore approaches, were also checked by an offshore LRFD method.

5.1 Storm characterisation

The Borkum West II turbine foundations had to be designed for a 50 year design storm. The build-up and dissipation of this 35 hour storm were based on guidance given by BSH (2011), resulting in the storm build-up chart shown in Figure 4. While NORSOK (2007) do not include wind speed in their recommendations, the wave height build-up and dissipation is the same as that shown in Figure 4.

Ten 600s coupled hydrodynamic/structural analyses were performed for each of the seven steps of the storm build-up profile to obtain time histories for the axial load variations applicable to the tripod foundation piles. Figure 5 shows one such time-history for the worst compression pile during the peak of the storm. Wave-induced loads led to the highest cyclic amplitudes. It should be noted that the Ultimate Limit State (ULS) event was included in one of the 600s time-histories during the peak of the storm.

Cyclic field pile and laboratory soil element tests conducted to assess the effects of cyclic loading on pile resistance usually apply regular amplitude cycles with fixed frequencies. However, as shown in Figure 5 the design load time-histories are composed of a succession of non-uniformly distributed irregular amplitude load cycles. In order to transform the design load time-histories into idealised series of uniform cycles with a given cyclic load amplitude (Q_{cyc}) and average load (Q_{avg}), as outlined schematically in Figure 6, significant peaks and troughs were identified in each of the seventy time-histories as indicated in Figure 5.

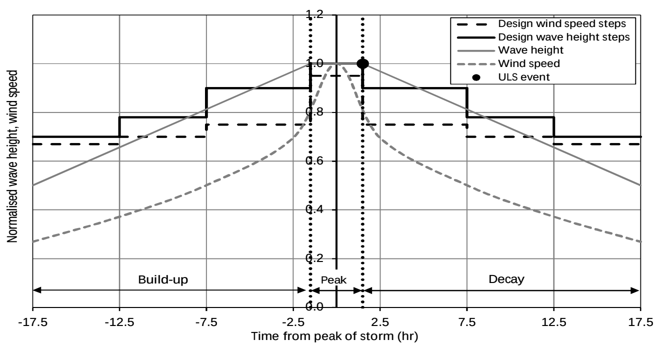

Figure 4: Storm build-up chart after BSH (2011).

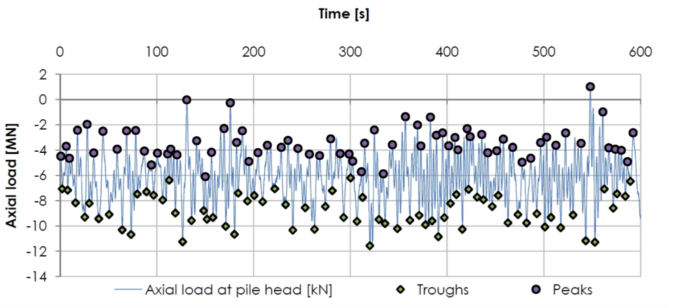

Figure 5: 600s time history for worst compression pile during peak of 35 hour design storm

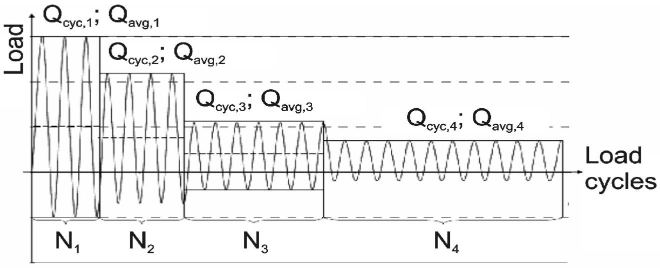

Figure 6: Idealised series of uniform load cycles (after BSH, 2011)

The 35 hour design storm could thus be discretised (accounting for the assumed length of each of the seven steps of the storm build-up chart indicated in Figure 4) into a series of idealised uniform load cycles as shown in Figure 7 for the worst tension pile. Figure 7 includes a total of nearly 15,000 load cycles.

5.2 Cyclic lateral load effects on axial resistance

Cyclic lateral loading of piles can reduce the axial and lateral stiffness of the surrounding soil over a certain depth below the ground surface. One expected consequence is reduced axial pile resistance within the zone of influence of the cyclic lateral loads. It should be noted that research into the effects of cyclic lateral loading on piles is active and new design methods are currently under development (e.g. Puech et al, 2012).

Recent research by Dührkop (2010) on one-way cyclic lateral loading of rigid monopiles in sands indicates that lateral cycling has a potentially significant effect over a zone of influence that extends to a depth of 2.6 pile diameters. Dührkop proposed a method for reducing the soil stiffness within this zone to account for the effects of cyclic lateral loading on the pile's lateral load-displacement response, using a reduction factor based on the magnitude of the cyclic load and the depth below the pile head. This methodology does not consider some of the parameters that are believed to be influential, including the detailed cyclic lateral loading history applied by storm conditions. However, the methodology is expected to be comparatively conservative for flexible tripod piles subject to two-way lateral cycling. Although many of the Borkum West II tripod piles have L/D ratios towards the lower limit of the usual range for offshore piles, it was considered reasonable in this case to adopt the Dührkop methodology to assess the response of the piles to cyclic lateral loading through a modified version of the conventional p-y analysis.

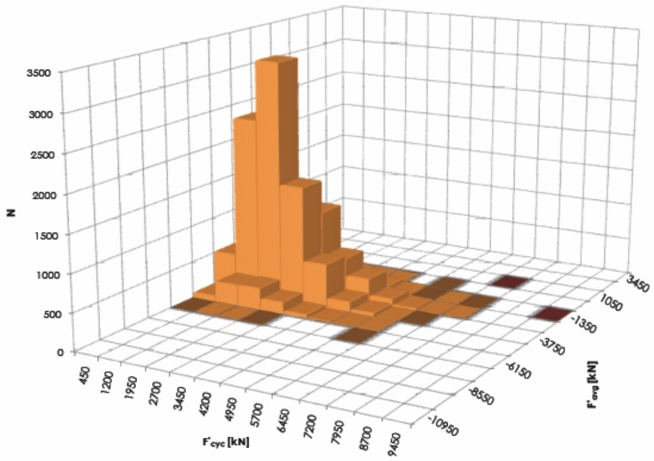

Figure 7: Histogram of uniform load cycles during 35 hour design storm for worst tension pile

It was also agreed during the design review that the lateral loading cycles could also affect the axial resistances. The factors adopted by Dührkop offered a reasonable engineering approach to represent the effects of cyclic lateral loading on the axial resistance of the piles. The same reduction factors were applied to the unit shaft friction within the zone of influence of cyclic lateral loads as follows:

$$t_1(z) = A(z)t_i(z)$$
$$A_{lat}(z) = \begin{cases} \dfrac{0.343}{0.9} \cdot \dfrac{z}{D} & \text{if } 0.343 \cdot \dfrac{z}{D} \leq 0.9 \\ 1 & \text{otherwise} \end{cases} \qquad (1)$$

where $A_{lat}(z)$ is the lateral reduction factor on the unit shaft friction, $t_1(z)$ is the reduced unit shaft friction distribution and $t_i(z)$ is the initial unit shaft friction distribution. The depth (z) is measured from the seabed level reduced by the design scour depth.

The above lateral reduction factors were applied to the static axial pile resistance calculated by the ICP method, before proceeding to review the degradation that might be caused by cyclic axial loading. Considering the turbine locations checked in the design review, lateral cyclic loading was estimated by this approach to reduce static axial shaft resistances by approximately 3 to 7%.

5.3 Cyclic axial load effects on axial resistance

When significant cyclic loading is applied, for example by storms, pile axial resistance can be degraded to values lower than the static resistance. Poulos (1988) proposed a cyclic stability chart that categorised the pile response as stable, meta-stable or unstable for given number of load cycles (N) as indicated schematically in Figure 8. The Borkum West II analysis was based on the effective stress approach set out by Jardine et al. (2005) for predicting the potential reduction of local shaft resistance for piles subject to groups of uniform load cycles in clays or sands. Relative losses in radial effective stress ($\Delta\sigma'_r/\sigma'_{r0}$) can be expressed as a function of the number of load cycles (N) and the normalised cyclic shear stress amplitude ($\tau_{cyc}/\tau_{\max\, stat}$), for example as:

$$\frac{\Delta s'_r}{s'_{r0}} = A\left(B + \frac{t_{cyc}}{t_{\max stat}}\right)N^C \quad (2)$$

Cyclic simple shear, CNS or triaxial tests may be undertaken to determine the material coefficients (A, B and C) and define rates of radial effective stress reduction under cycling using Equation (2) as in the Clair project (|Jardine et al 2011). As no such data was available for Borkum West II, reference was made to the Dunkirk field tests on piles driven in North Sea sand reported by Jardine & Standing (2000).

During the review process, a simplified global approach was developed recognising that the average loss of radial effective stress determines the loss of static shaft resistance (ΔQ_{stat}). The relative loss of shaft resistance ($\Delta Q_{stat}/Q_{\max\, stat}$) was expressed as a function of the normalised cyclic load amplitude ($Q_{cyc}/Q_{\max\, stat}$) as follows:

$$\frac{\Delta Q_{stat}}{Q_{\max stat}} = A\left(B + \frac{Q_{cyc}}{Q_{\max stat}}\right)N^C \quad (3)$$

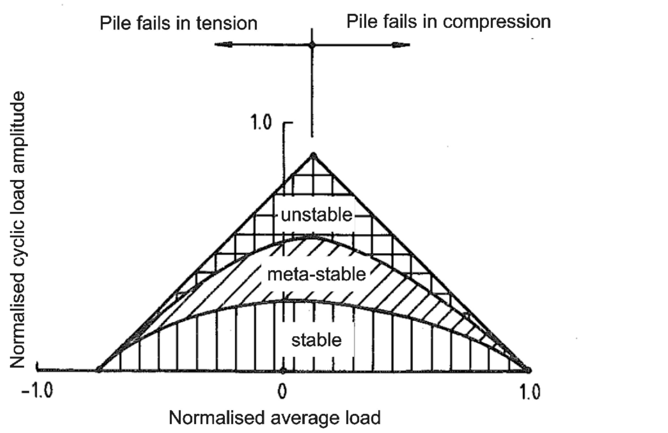

Figure 8: Schematic representation of cyclic stability diagram after Poulos (1988)

Direct calibration with the Dunkirk test results (Jardine & Standing, 2000) gives the following values:

$$A = -0.126;\ B = -0.100,\ C = 0.45$$

Using Equation (3) the interaction diagram shown in Figure 9 was developed, with lines of constant N_f (the number of cycles to failure for a given load amplitude) being very similar to those reported by Jardine et al (2005). Also shown in Figure 9 is a limit to $Q_{cyc}/Q_{\max\, stat}$ below which it is assumed that cycling does not degrade pile resistance. This cyclic damage limit has been defined as follows:

$$\frac{Q_{cyc}}{Q_{\max stat}} < 0.3\left(1 - \frac{Q_{avg}}{Q_{\max stat}}\right) \quad (4)$$

As proposed by Poulos (1988), and proven in field tests at Dunkirk (Jardine & Standing, 2000) and model tests (Richter et al 2010 and Rimoy et al 2012), cycling applied below certain levels can lead to both a fully stable response and potential increases in shaft resistance.

The global approach does not explicitly capture the local mechanisms involved in the reduction of pile resistance due to cyclic axial loading (see for example Rimoy et al 2012). It is acknowledged that, for example, shaft friction reduction is not homogeneous, but propagates from the pile head to its toe under increasing severity of the cyclic loading and number of cycles with the flexibility of the pile determining the distribution of friction along the pile shaft. If required, the expression for the local reduction in radial effective stress (Equation 2) can be input into T-Z formulations which have been shown to reproduce the Dunkirk full scale pile tests. Such an approach has been applied in offshore soil-structure

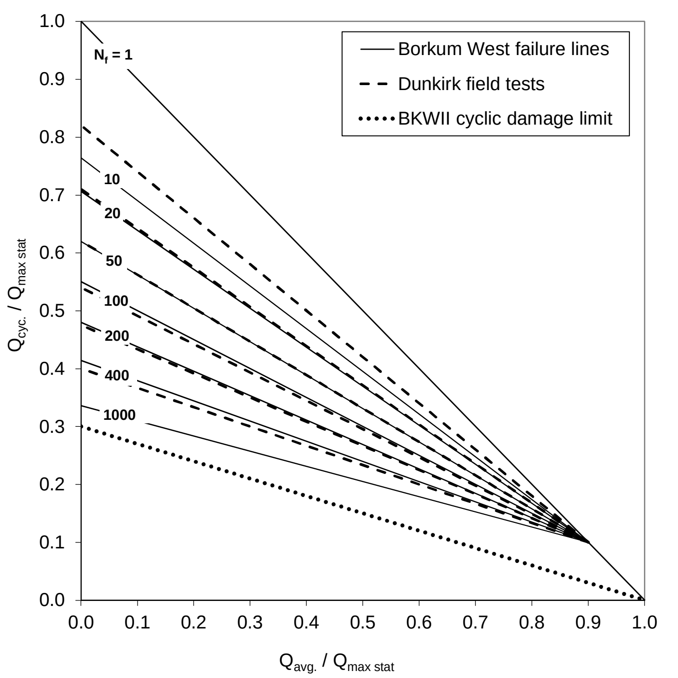

Figure 9: Cyclic interaction diagram

interaction analyses (e.g. Atkins, 2000). The code CATZ developed by CA was applied to perform calculations of this type at selected Borkum West II pile locations.

The losses of static shaft resistance (ΔQ_{stat}) expected during each of the uniform groups of load cycles lying above the cyclic damage limit can be assessed using Equation (3). The ultimate overall impact of the design storm can then be estimated by applying an equivalent cycle method effectively 'curve hopping' between groups of load cycles. This treatment implicitly assumes that Miner's hypothesis holds (Miner, 1945). While this has not been verified comprehensively for soils, and little is known regarding load history effects, the approach was able to reasonably predict the impact of multi-stage tests on the Dunkirk driven piles (Atkins, 2000).

The discretisation of the design storm resulted in a total of nearly 15,000 uniform load cycles in groups with given cyclic load amplitudes and average loads (see Figure 7). In order to determine which load cycles required assessment, the cyclic load amplitude and average load for each group were normalised by the corresponding static shaft resistance ($Q_{\max\, stat}$) and compared with the cyclic damage limit. Depending on the value of $Q_{\max\, stat}$ determined for each turbine location, different groups of uniform load cycles needed to be included in calculating the decrease in axial shaft resistance in response to axial cyclic loading. At locations with a relatively large axial shaft resistance, such as in Figure 3(b), no load cycles lay above the cyclic damage limit and no reduction of shaft resistance was predicted. At locations with lower axial shaft resistance, such as in Figure 3(a), up to around 100 load cycles (out of a total of nearly 15,000) lay above the cyclic damage limit. At locations where damaging cyclic axial loads occurred, their combined effect was estimated to degrade the shaft resistance (on top of lateral cyclic load reductions) by about 10 to 25%. Broadly similar results were obtained by the CA CATZ analysis.

5.4 Design approach

The design criterion adopted for Borkum West II in relation to axial pile load resistance, accounting for cyclic load reductions, was based on the Eurocode methodology and terminology set out by Richter et al. (2010) using the following governing inequality:

$$\sum E_{d,i} \le \sum R_{d,i} - g_Q h_{R,d} \Delta R_k \qquad (3)$$

where:

- $\Sigma E_{d,i}$ is the sum of the design (factored) actions (using a partial factor $\gamma_f = 1.35$)
- $\Sigma R_{d,i}$ is the design (factored) value of the axial pile resistance (using a partial factor $\gamma_R = 1.4$)
- γ_Q is the partial safety factor for unfavourable load conditions ($\gamma_Q = 1.5$)
- $\eta_{R,d}$ is a model factor for the determination of $\Delta R_{,k}$ ($\eta_{R,d} = 1.2$)
- ΔR_k is the characteristic reduction in pile resistance due to cyclic loading

This inequality defines the design criterion as requiring the sum of the design actions to be less than or equal to the sum of the design resistance, minus the design (factored) value of the reduction in pile resistance due to cyclic loading.

The design actions were derived from the single extreme ULS event occurring during the peak of the 50 year design storm, making allowances for uplift and out-of-verticality loads. The axial pile resistance was calculated as described above. For the tension case, the weight of the pile and the soil plug were included, albeit using a partial factor, $\gamma_R = 1.0$.

When applying this inequality during the design review, pile shaft resistance values (reduced after lateral cyclic loading effects) were used in the $\Sigma R_{d,i}$ term. The reductions in pile resistance due to axial cyclic loading effects were thus included only in the ΔR_k term. Richter et al. (2010) do not discuss the effects of lateral cyclic loading, and therefore the effects of lateral cyclic loading could also be included in the ΔR_k term rather than the $\Sigma R_{d,i}$ term. Based on this approach, the review identified some cases with minor shortfalls in tension resistance, leading to minor extensions of the design pile lengths.

Offshore oil and gas designers often apply reliability based LRFD (Load and Resistance Factor Design) approaches. Using an LRFD approach with the load factors suggested by Jardine et al. (2005) for 'unmanned' structures indicates that the original pile designs for Borkum West II were generally sufficient. It should be noted that the latter LRFD factors are more onerous than those recommended by API (2011).

6. Conclusions

Key aspects of the pile design methodology for the Borkum West II wind farm developed during a collaborative technical review process are reported in this paper. In addition to reassessing the site investigation data and the static pile resistance design, the review considered pile driving risks and the assessment of cyclic loading effects.

The high quality site investigation data allowed reliable application of the ICP design method, with independent calculations showing generally good agreement of static axial pile resistances between different parties. Lateral loading analyses also indicated satisfactory performance after allowing for negative effects of cyclic loading. The pile driving risk assessment led to wall thickness increases at several turbine locations to minimise the risk of tip buckling in very dense sands.

A detailed characterisation of the design storm and assessment of cyclic loading effects showed that piles with relatively large static axial shaft resistances were unlikely to degrade under storm loading. However, for turbine locations with relatively low static axial resistances shaft capacity reductions of up to about 25% were calculated. Application of the chosen design methodology led to some pile lengths being extended marginally. Checks involving an LRFD approach generally demonstrated satisfactory final pile designs.

7. References

Alm, T., Snell, R.O., Hampson, K.M. & Olaussen, A. (2004). Design and installation of the Valhall piggyback structures. *Proc. 36th Offshore Technology Conference.* Houston, Paper OTC 16294.

API (2011). Geotechnical and foundation design considerations, ANSI/API Recommended Practice 2GEO, First edition

Atkins (2000). *Cyclic degradation of offshore piles*. Offshore Technology Report OTO 2000/013. Health and Safety Executive, London.

BSH (2007). Standard. Design of Offshore Wind Turbines. Bundesamt für Seeschifffahrt und Hydrographie, 20 December 2007.

BSH (2008). Standard. Ground Investigations for Offshore Wind Farms. 25 February 2008.

BSH (2011). Guidance for use of the BSH standard “Design of Offshore Wind Turbines”.

Dührkop, J. (2010). Zum Einfluss von Aufweitungen und zyklischen Lasten auf das Verformungsverhalten lateral beanspruchter Pfähle in Sand, in: *TU Hamburg, Veröffentlichungen des Instituts für Geotechnik und Baubetrieb*, Heft 20 (in German).

Evans T.G., Finnie, I., Little, R., Jardine R.J. & Aldridge, T.R. (2010). ‘BP Clair Phase 1 Design and assurance of driven piled foundations in extremely hard till’, *2nd Int. Symp. on Frontiers in Offshore Geotechnics*, Perth, Australia.

MSL Engineering Ltd (2001). *A study of pile fatigue during driving and in-service and of pile tip integrity*. Offshore Technology Report 2001/018. HSE, London.

Ho, Y. K., Jardine, R.J. & Anh-Minh, N. (2010). Large displacement interface shear between steel & granular media. *Geotechnique*, 61(3): 221-234.

Jardine, R.J. (1994) Review of offshore pile design for cyclic loading: North Sea clays. HSE Offshore Technology Report, OTN 94 157.85.

Jardine, R. J. and Standing, J.R. (2000). Pile load testing performed for HSE cyclic loading study at Dunkirk, France. 2 Vols. Offshore Technology Report OTO 2000 007. HSE, London.

Jardine, R.J., Chow, F.C., Overy, R. and Standing, J.R. (2005). *ICP Design Methods for Driven Piles in Sands and Clays*. Thomas Telford Publishing.

Jardine, R.J. (2009). Review of technical issues relating to foundations and geotechnics for offshore installations in the UKCS. Research Report RR676. HSE, London.

Jardine, R.J., Aldridge, T. & Evans, T.G. (2011). Offshore foundation engineering in extremely dense glacial tills West of the Shetland Islands. *Proc. 15th ECSMGE*, Athens, IOS Press, Amsterdam, Vol. 2, 879-884.

Miner, M.A. (1945). Cumulative damage in fatigue. *Journal of Applied Mechanics* 12 (1945): A159–A164.

NORSOK (2007). Action and action effects. Standard N-003. Edition 2, September 2007

Poulos, H.G. (1988). Cyclic stability diagram for axially loaded piles. ASCE J. Geotech. Engrg. 114(8): 877-895.

Puech, A., Canou, J., Bernardini, C., Pecker, A., Jardine, R. & Holeyman, A. (2012). SOLCYP: A four year JIP on the behaviour of piles under cyclic loading. *Offshore Site Investigation and Geotechnics*, SUT, London.

Randolph, M., Cassidy, M., Gourvenec, S and Erbrich, C. (2005). Challenges of offshore geotechnical engineering. *Proc. 16th ICSMGE*, Osaka, Millpress, Rotterdam, Vol. 1, 123–176.

Richter, T., Kirsch, F. & Mittag, J. (2010). Bemessungskonzepte für axial-zyklisch belastete Pfähle – Ein Überblick und neue Ansätze. Tagungsband zur 31. Baugrundtagung der DGGT in München, 263-269 (in German).

Rimoy, S., Jardine, R., Silva, M., Foray, P., Tsuha, C. & Yang, Z. (2012). Local and global behaviour of axial cyclic loaded instrumented model displacement piles in sand. *Offshore Site Investigation and Geotechnics*, SUT, London.